Chapter 5
Test Results and Discussion

The test results of the seven beams described in Chapter 3 are presented in the following chapter. Their behavior throughout the static test to failure is described using recorded data, observed crack patterns, and the mode of failure. Also, the behavior of each beam is compared to that predicted by the model presented in Chapter 4.

5.1 Test Results

For each beam, the results of the tests are presented. Along with the test details; the recorded data, crack patterns, and mode of failure are discussed. Also, the predicted behavior of each beam, as discussed in Chapter 4, is compared to the experimental results. Due to complications of measurement devices, the steel reinforcement strains will be excluded from this section; however, these results are shown in Appendix D.

5.1.1 Control Beam

The control beam was tested in order to have an unstrengthened beam to compare to the CFRP strengthened beams. Stroke control in increments of 1000 pounds was used in loading the specimen. After each load increment, the beam was inspected for cracks and any possible failure signs. Figure 5.1 shows the moment-deflection plot for the actual beam and the moment-deflection plot obtained from the theoretical model. The beam appeared to display linear behavior to the cracking moment and from that point to first yield of the steel reinforcement. After yielding of the reinforcement began, a large increase in deflection was noticed, while the applied moment changed little. This behavior continued until failure was caused by crushing of the concrete at the top of the beam, similar to Figure 5.2. The test was terminated once crushing of the concrete began.

No cracks were noticed in the control beam until one formed at the crack propagator at mid-span. Afterwards, small cracks began developing vertically in the constant moment region. They were similar in width, but displayed no distinct spacing pattern. When the tension reinforcement began to yield, shear and flexural-shear cracks began forming in the shear region of the beam. All cracks continued to grow in both width and length while the direction of their travel started to lean towards the load and support points at approximately 45° angles. This continued until the cracks reached approximately 85% of the beam depth and failure occurred.

The theoretical model accurately predicted the behavior of the test beam. The test results at the points where the steel reinforcement began to yield and at the maximum nominal capacity are compared to the theoretical model in Table 5.1.

Table 5.1: Tested and Theoretical Properties of Control Beam

	Yielding Moment, in-kips	Yielding Deflection, in.	Maximum Moment, in-kips	Maximum Deflection, in.	Mode of Failure
Tested	480	0.83	564	1.95	Crushing of Concrete
Theoretical	505	0.72	543	1.83	Crushing of Concrete
Percent Difference	5	13	4	6	N/A

The only significant difference between the actual data and theoretical prediction was the deflection at yielding of the steel reinforcement. The model under-predicted this test value by 13%, while the other critical test values differed from the model by 6% or less. Also, as shown in Figure 5.1, the theoretical model predicted a slightly stiffer elastic portion of the moment-deflection plot.

5.1.2 Two-Layer, 0°/90° Strengthened Beam

The two-layer, 0°/90° strengthened specimen was tested using load control in increments of 1000 pounds. Crack patterns and possible failure signs were investigated after each load increment. Figure 5.3 shows the moment-deflection plots for the actual beam and from the theoretical model. The beam appeared to have linear behavior until the cracking moment was reached. After this point, the linear behavior continued, but the stiffness was reduced. After yielding of the steel reinforcement began, cracking sounds were heard which could be an indication of the CFRP debonding from the concrete. The beam continued to deflect with little increase in moment resistance until the CFRP ruptured, as shown in Figure 5.4. The failure of the CFRP was quick and explosive. After the CFRP ruptured, the damaged beam continued to resist about 80% of the largest moment resistance until the concrete at the top of the beam began to crush. This behavior is not shown in Figure 5.3, due to the loss of instrumentation after the rupture of the CFRP. Since the damaged beam was never able to reach the largest moment resistance that occurred before the CFRP ruptured, the beam failure is attributed to the rupture of the CFRP. The test was terminated once the crushing began at the top fiber of the beam.

The first crack for the two-layer, 0°/90° composite beam was observed at the crack propagator. Next, small, vertical cracks began to form in the constant moment region. They continued to travel up the depth of the beam until yielding of the steel reinforcement began. After this point, more flexural cracks formed throughout the beam, as well as, shear-flexural cracks in the lower-moment regions. Unlike the control beam, the cracks were smaller in width and evenly spaced. Most cracks traveled approximately 50% of the beam depth before failure occurred. The exception was a few larger cracks that formed in the constant moment region. The larger cracks reached about 80% of the beam depth before the CFRP ruptured directly below a large crack.

The theoretical model accurately predicted the behavior of the test beam. The test results at the points where the steel reinforcement began to yield and at the maximum nominal capacity are compared to the theoretical model in Table 5.2.

Table 5.2: Tested and Theoretical Properties of Two-Layer, 0°/90° Strengthened Beam

	Yielding Moment, in-kips	Yielding Deflection, in.	Maximum Moment, in-kips	Maximum Deflection, in.	Mode of Failure
Tested	547	0.87	611	1.60	CFRP Rupture
Theoretical	530	0.73	616	1.65	Crushing of Concrete
Percent Difference	3	14	1	3	N/A

The only significant difference between the actual data and theoretical prediction was the deflection at yielding of the steel reinforcement. The model under-predicted this test value by 14%, while the other critical test values differed from the model by 3% or less. Also, as shown in Figure 5.4, the theoretical model predicted a slightly stiffer elastic portion of the moment-deflection plot.

Figure 5.5 shows the moment-CFRP strain plots for both the experimental beam and the theoretical model. The model accurately predicted both linear portions of this plot. However, the CFRP ruptured at a much lower strain (0.008 in/in) than was seen with the uniaxial test samples (0.012 in/in) discussed in Chapter 3. This led to an inaccurate prediction of the failure mode, since the model predicted the concrete crushing in the extreme compression fiber before the CFRP reached the average failure strain of the uniaxial test samples.

5.1.3 Three-Layer, 0°/90° Strengthened Beam

The three-layer, 0°/90° strengthened specimen was tested using stroke control in increments of 1000 pounds. Crack patterns and possible failure signs were observed after each load increment. Unlike the other composite specimens, there was a flaw with the CFRP application process for this beam. As discussed in Chapter 3, the layers of CFRP were not completely bonded to the concrete beam, which caused gaps between the concrete and CFRP in the constant moment region. Figure 5.6 shows the moment-deflection plots for the actual beam and from the theoretical model. The beam appeared to display linear behavior to the point at which cracking moment occurred, and continued from there until the yielding of the steel reinforcement began. After the steel began to yield, cracking noises were heard. Also, all gaps between the CFRP and concrete beam, that were initially present, closed. Next, rupture of the CFRP occurred, similar to Figure 5.4, and was accompanied by a sudden, explosive sound. Although the loss of instrumentation caused behavior after the failure to be absent from Figure 5.6, load continued to be applied to the damaged beam after the CFRP ruptured. The damaged beam was able to sustain about 80% of the largest moment resistance before crushing of the concrete in the extreme compression fiber occurred. Since the beam reached the largest moment resistance before the CFRP ruptured, the beam failure is attributed to the rupture of the CFRP. The test was stopped after the crushing of the concrete began.

The cracking of the three-layer, 0°/90° composite beam began with a crack above the propagator. Next, hairline cracks started forming vertically in the constant moment region. These continued to grow in width and length until the steel reinforcement began to yield. After this point, shear and shear-flexural cracking formed throughout the beam. The flexural cracks in the constant moment region grew in width and were not evenly spaced. The cracks reached about 80% of the beam depth before rupture of the CFRP took place under a considerably large crack in the constant moment region.

The theoretical model accurately predicted the behavior of the test beam. The test results at the points where the steel reinforcement began to yield and at the maximum nominal capacity are compared to the theoretical model in Table 5.3.

Table 5.3: Tested and Theoretical Properties of Three-Layer, 0°/90° Strengthened Beam

	Yielding Moment, in-kips	Yielding Deflection, in.	Maximum Moment, in-kips	Maximum Deflection, in.	Mode of Failure
Tested	546	0.86	634	1.57	CFRP Rupture
Theoretical	544	0.73	647	1.58	Crushing of Concrete
Percent Difference	1	15	2	1	N/A

The only significant difference between the actual data and theoretical prediction was the deflection at yielding of the steel reinforcement. The model under-predicted this test value by 15%, while the other critical test values differed from the model by 2% or less. Also, as shown in Figure 5.6, the theoretical model predicted a slightly stiffer elastic portion of the moment-deflection plot.

Figure 5.7 shows the moment-CFRP strain plots for both the experimental beam and the theoretical model. The CFRP failed due to rupture at a strain of 0.016 in/in, which was double the predicted strain of 0.008 in/in, which led to an inaccurate prediction of the failure mode. The unexpected CFRP strain behavior of the three-layer, 0°/90° composite beam was due to the improper bond between the CFRP and concrete caused by the flaw in the application process. Since the CFRP was not bonded properly, portions of the CFRP were not completely confined to the concrete beam. This allowed the CFRP to reach a much larger strains than if the CFRP was completely confined to the concrete beam.

5.1.4 Four-Layer, 0°/90° Strengthened Beam

The four-layer, 0°/90° strengthened specimen was tested with load control in increments of 1000 pounds. After each load increment, the beam was investigated for cracks and any possible failure signs. Figure 5.8 shows the moment-deflection plot for the experimental beam and from the theoretical model. The beam experienced linear behavior to the cracking moment and from that point to first yielding of the steel reinforcement. Cracking sounds were heard frequently after yielding of the reinforcement began. Towards the end of the test, crushing of the concrete in the extreme compression fiber was noticed, similar to Figure 5.2. Although not seen in Figure 5.8, the damaged beam continued to hold about 90% of the largest moment resistance after the concrete began to crush. Suddenly, the CFRP ruptured with an explosive sound, and the beam experienced a catastrophic failure. Since the crushing of the concrete preceded the CFRP rupture, the concrete beam was already damaged. Therefore, after the CFRP ruptured, the compressive steel reinforcement buckled and the concrete began to crumble rapidly. The load had to be retracted immediately, and the test was terminated. The failure of the beam was attributed to the crushing of the concrete since the largest moment the beam resisted occurred immediately before crushing of the concrete began.

The cracking of the four-layer, 0°/90° composite beam began with a crack above the crack propagator. Next, small, vertical cracks began to form in the constant moment region. They continued to travel up the depth of the beam until yielding of the steel reinforcement began. At this point, shear-flexural and more flexural cracks began forming throughout the beam. The smaller cracks seemed evenly spaced, and most of them traveled approximately 60% of the beam depth before failure occurred. The exception was a few larger cracks that formed in the constant moment region. The larger cracks reached about 90% of the beam depth before the concrete began crushing at the extreme compression fiber.

The theoretical model accurately predicted the behavior of the test beam. The test results at the points where the steel reinforcement began to yield and at the maximum nominal capacity are compared to the theoretical model in Table 5.4.

Table 5.4: Tested and Theoretical Properties of Four-Layer, 0°/90° Strengthened Beam

	Yielding Moment, in-kips	Yielding Deflection, in.	Maximum Moment, in-kips	Maximum Deflection, in.	Mode of Failure
Tested	548	0.86	670	1.70	Crushing of Concrete
Theoretical	555	0.73	674	1.52	Crushing of Concrete
Percent Difference	1	15	1	11	N/A

The critical moments of the test results were predicted accurately. However, the deflection at yielding of the steel reinforcement and the maximum deflection were under-predicted by the model by 15% and 11%, respectively. Also, as shown in Figure 5.8, the theoretical model predicted a slightly stiffer elastic portion of the moment-deflection plot.

The moment-CFRP strain plots for both the experimental beam and the theoretical model are shown in Figure 5.9. Comparing the plots, the slopes are approximately the same throughout. However, the ultimate strain experienced by the CFRP was slightly smaller than the predicted value.

5.1.5 Two-Layer, ±45° Strengthened Beam

The two-layer, ±45° strengthened specimen was tested in increments of approximately 1000 pounds with load being controlled by hydraulic pressure. After each load increment, the beam was observed for cracks and any possible failure signs. As discussed in Chapter 3, a flaw occurred in the bonding process of the CFRP to the beam. The CFRP layers slipped when the bonding load was applied, which caused the CFRP to be off-center and positioned on one edge of the concrete beam. Figure 5.10 shows the moment-deflection plots for the experimental beam and from the theoretical model. The beam appeared to have linear behavior to its cracking moment, as well as, from cracking moment to the point at which the steel reinforcement began to yield. About the same time the reinforcement began to yield, cracking was heard followed by a popping sound. Upon inspection, the CFRP appeared to debond in the constant moment region, as shown in Figure 5.11. It was noticed only on the edge of the beam that was closer to the CFRP layers. However, as the CFRP continued to debond down the length of the beam, the deflection and moment kept increasing. This continued until the concrete began to crush at the extreme compression fiber. Since the moment continued to increase after the CFRP debonded, the mode of failure was attributed to the crushing of the concrete. The debonding of the CFRP reached approximately three-quarters of the beam length, but never ruptured. The test was terminated after the crushing of the concrete began.

The cracking of the two-layer, ±45° composite beam began with a crack above the propagator. Next, small, vertical cracks began to form in the constant moment region. They continued to travel up the depth of the beam until yielding of the steel reinforcement began. Next, shear-flexural and more flexural cracks formed throughout the beam and traveled towards the load points and supports. The spacing and size of the cracks were very similar to the cracking pattern of the control beam. The cracks traveled approximately 85% of the beam depth before failure occurred.

The behavior of the beam differed from what the theoretical model predicted. The test results at the points where the steel reinforcement began to yield and at the maximum nominal capacity are compared to the theoretical model in Table 5.5.

Table 5.5: Tested and Theoretical Properties of Two-Layer, ±45° Strengthened Beam

	Yielding Moment, in-kips	Yielding Deflection, in.	Maximum Moment, in-kips	Maximum Deflection, in.	Mode of Failure
Tested	491	0.90	552	1.81	Crushing of Concrete
Theoretical	518	0.71	581	1.73	Crushing of Concrete
Percent Difference	5	21	5	5	N/A

The model overestimated the critical moments by 5%, and underestimated the deflections by as much as 21%. This was caused by the CFRP not providing any added strength or stiffness to the beam after the material began to debond. Also, as shown in Figure 5.10, the theoretical model predicted a slightly stiffer elastic portion of the moment-deflection plot.

The moment-CFRP strain plot for both the experimental beam and the theoretical model is shown in Figure 5.12. Comparing the plots, the maximum CFRP strain was 0.0125 in/in, while the predicted value was 0.0092 in/in. This large difference could be attributed to the debonding of the CFRP, which allowed the unconfined CFRP to reach larger strain values than predicted.

5.1.6 Three-Layer, ±45° Strengthened Beam

The three-layer, ±45° strengthened specimen was tested with load control in increments of 1000 pounds. After each load increment, the beam was inspected for cracks and any possible failure signs. Figure 5.13 shows the moment-deflection plot for the tested beam and from the theoretical model. The beam seemed to have linear behavior until the steel reinforcement began to yield. No change in stiffness was observed at the cracking moment. After the steel began to yield, cracking sounds were heard and the CFRP debonded from the concrete beam, as shown in Figure 5.14. However, unlike the two- layer, ±45° composite beam, the CFRP layers shifted away and separated from the beam in the constant moment region. This was caused by aggregate interlock at the shear interface between the CFRP and concrete beam after debonding. Following the debonding of the CFRP, the moment and deflection continued to increase until crushing of the concrete in the extreme compression fiber occurred. The debonding of the CFRP was only observed in the constant moment region and the CFRP did not rupture. The test was terminated after the crushing of the concrete began.

The cracking of the three-layer, ±45° composite beam began with a crack at the crack propagator. Next, small, vertical cracks began to form in the constant moment region. They continued to travel up the depth of the beam until yielding of the steel reinforcement began. After this point, shear-flexural and more flexural cracks formed throughout the beam. The cracks were evenly spaced, and number was almost double that of the control beam. A majority of the cracks traveled approximately 70% of the beam depth before failure occurred.

The behavior of the beam differed from what the theoretical model predicted. The test results at the points where the steel reinforcement began to yield and at the maximum nominal capacity are compared to the theoretical model in Table 5.6.

Table 5.6: Tested and Theoretical Properties of Three-Layer, ±45° Strengthened Beam

	Yielding Moment, in-kips	Yielding Deflection, in.	Maximum Moment, in-kips	Maximum Deflection, in.	Mode of Failure
Tested	534	0.80	596	1.82	Crushing of Concrete
Theoretical	526	0.71	600	1.68	Crushing of Concrete
Percent Difference	2	11	1	8	N/A

The model accurately predicts the critical moments. However, the yield and ultimate deflections differ from the model by 11% and 8%, respectively. Debonding of the CFRP contributed to this inaccuracy, since it was not accounted for in the theoretical model. However, as shown in Figure 5.14, the stiffness of the tested beam and theoretical model are similar. This implies this type of CFRP debonding had little effect on the stiffness of the strengthened beam.

The moment-CFRP strain plots for the experimental beam and the theoretical model are shown in Figure 5.15. Comparing the plots, the slope of the linear segment of the test results was much larger than the theoretical model predicted. Also, the maximum strain of the CFRP for the tested beam was 0.008 in/in, which was slightly smaller than the predicted strain of 0.009 in/in.

5.1.7 Four-Layer, ±45° Strengthened Beam

The four-layer, ±45° strengthened specimen was tested using load control in increments of 1000 pounds. After each load increment, the beam was inspected for cracks and any possible failure signs. Figure 5.16 shows the moment-deflection plots for the experimental beam and from the theoretical model. The behavior of the specimen is very similar to the three-layer, ±45° composite beam. The beam appeared to have linear behavior until the steel reinforcement began to yield. No change in stiffness was observed at the cracking moment. After the steel began to yield, cracking sounds were heard and the CFRP debonded from the concrete beam in the same manner as the three-layer, ±45° composite beam (shown in Figure 5.14). After the CFRP debonded, the moment and deflection continued to increase until crushing of the concrete in the extreme compression fiber occurred. . The debonding of the CFRP was only observed in the constant moment region and the CFRP never ruptured. The test was terminated after the crushing of the concrete began.

The cracking of the four-layer, ±45° composite beam began with a crack at the crack propagator. Next, small, vertical cracks began to form in the constant moment region. They continued to travel up the depth of the beam until yielding of the steel reinforcement began. After this point, shear-flexural and more flexural cracks formed throughout the beam. The cracks were evenly spaced, and number was almost double that of the control beam. Most of the cracks traveled approximately 70% of the beam depth before failure occurred.

The behavior of the beam differed from what the theoretical model predicted. The test results at the points where the steel reinforcement began to yield and at the maximum nominal capacity are compared to the theoretical model in Table 5.7.

Table 5.7: Tested and Theoretical Properties of Four-Layer, ±45° Strengthened Beam

	Yielding Moment, in-kips	Yielding Deflection, in.	Maximum Moment, in-kips	Maximum Deflection, in.	Mode of Failure
Tested	541	0. 81	632	2.05	Crushing of Concrete
Theoretical	532	0.71	618	1.64	Crushing of Concrete
Percent Difference	2	12	2	20	N/A

The model accurately predicts the critical moments. However, the tested yield and maximum deflections differ from the theoretical model by 12% and 20%, respectively. As previously explained with the three-layer, ±45° strengthened beam, debonding of the CFRP contributed to an inaccuracy in deflection, since it was not accounted for in the theoretical model. However, as shown in Figure 5.16, the stiffness of the tested beam and theoretical model were similar implying this type of CFRP debonding had little effect on the stiffness of the strengthened beam.

The moment-CFRP strain plots for the experimental beam and from the theoretical model are shown in Figure 5.17. The slope of the test results is slightly larger than the theoretical model prediction. Also, the maximum CFRP strain reached 0.0125 in/in, while the predicted value was 0.009 in/in. This large difference could be attributed to the debonding of the CFRP, which allowed the unconfined CFRP to reach larger strain values than predicted.

5.1.8 Summary

A summary of the tested and theoretical properties for the control and CFRP strengthened beams are shown in Table 5.8.

Table 5.8: Summary of Tested and Theoretical Properties

Test Beams	Control	Two-Layer, 0°/90°	Three-Layer, 0°/90°	Four-Layer, 0°/90°	Two-Layer, ±45°	Three-Layer, ±45°	Four-Layer, ±45°
Tested Yielding Moment, in-kips	480	547	546	548	491	534	541
Theoretical Yielding Moment, in-kips	505	530	544	555	518	526	532
Percent Difference	5	3	1	1	5	2	2
Tested Yielding Deflection, in.	0.83	0.87	0.86	0.86	0.90	0.80	0.81
Theoretical Yielding Deflection, in.	0.72	0.73	0.73	0.73	0.71	0.71	0.71
Percent Difference	13	14	15	15	21	11	12
Tested Maximum Moment, in-kips	564	611	634	670	552	596	632
Theoretical Maximum Moment, in-kips	543	616	647	674	581	600	618
Percent Difference	4	1	2	1	5	1	2
Tested Maximum Deflection, in.	1.95	1.60	1.57	1.70	1.81	1.82	2.05
Theoretical Maximum Deflection, in.	1.83	1.65	1.58	1.52	1.73	1.68	1.64
Percent Difference	6	3	1	11	5	8	20

As seen by the table, the moments at first yield of the steel reinforcement and at maximum capacity for every test beam is reasonably predicted by the theoretical model. The largest percent difference between the moment of the test beam and the theoretical model is 5%. However, the deflections do not correspond to the theoretical predictions as well as the moments. The largest percent difference between deflection of the test beam and the theoretical model is 20%. Due to the unforeseen debonding of the CFRP from the concrete beam, the inaccuracy of the maximum deflections is more evident with the ±45° strengthened beams. However, the deflection at yielding of the steel reinforcement was under-predicted by the theoretical model for every specimen by 13%-21%. This inaccuracy might have been due to the assumption that 29000 ksi was the modulus of elasticity for the steel reinforcement. This property is critical in the theoretical model to accurately predict the deflection at yielding of the steel reinforcement. If the actual modulus of the steel reinforcement was less than 29000 ksi, the deflection at yielding of the steel reinforcement would have changed without significantly altering the other predicted values.

5.2 Analysis of Test Results

The following section compares the strength and ductility of the strengthened beams for each CFRP orientation (0°/90° and ±45°). The maximum strengths of each tested beam are related to investigate a pattern between strength and the number of CFRP layers applied for each fiber orientation. Also, using the two methods discussed in Chapter 4, the ductility of each tested beam is compared to the theoretical predictions.

5.2.1 Flexural Strength

The maximum strength of each tested beam and the percent increase in strength with respect to the control beam is shown in Table 5.9.

Table 5.9: Tested Maximum Moment Comparison of Strengthened Beams to Control Beam

Test Beams	Maximum Moment, in-kips	Percent Increase w/r to Control Beam
Control	564	N/A
Two-Layer, 0°/90°	611	8
Three-Layer, 0°/90°	634	12
Four-Layer, 0°/90°	670	19
Two-Layer, ±45°	552	-2
Three-Layer, ±45°	596	6
Four-Layer, ±45°	632	12

The 0°/90° strengthened beams have a larger and more proportional increase in strength per layer than the ±45° strengthened beams. If the percent increase of the 0°/90° strengthened beams are divided by the number of layer applied, an increase in strength per layer of 4.2%, 4.1%, and 4.7% are shown for the two-layer, three-layer, and four-layer strengthened beams, respectively. This indicates a close association between the number of layers applied and the increase in strength. However, the relationship between the ±45° strengthened beams is not as precise as the 0°/90° strengthened beams. If the percent increase of each ±45° strengthened beam is divided by the number of layer applied, an increase in strength per layer of –1.1%, 1.9%, and 3.0% are shown for the two-layer, three-layer, and four-layer strengthened beams, respectively. This indicates the number of ±45° CFRP layers applied to the beams does not correspond with the increase in strength. In fact, the two-layer beam actually shows a decrease in strength with respect to the control beam (due to a flaw in the bonding procedure).

The test plots for the 0°/90°and ±45° CFRP strengthened beams are shown in Figure 5.18 and 5.19, respectively. The plots indicate similar behavior of the strengthened beams until yielding of the reinforcement steel begins, at which time, the specimens increase in strength proportional to the number of CFRP layer applied. However, due to the slip of the CFRP layers during bonding (as discussed earlier), the two-layer, ±45°composite beam did not act in this manner, but instead, behaved similar to the control beam.

5.2.2 Ductility

The ductility of the test specimens is analyzed using the conventional and energy methods discussed in Chapter 4. Also, the theoretical and tested ductility indexes are compared with each other to investigate the accuracy of the predicted value.

Table 5.10 shows a comparison between the conventional and energy ductility indexes measured for each tested beam.

Table 5.10: Comparison of Conventional and Energy Ductility Index for Test Specimens

Test Beams	Conventional Ductility Index	Energy Ductility Index	Percent Difference
Control	2.34	2.29	2
Two-Layer, 0°/90°	1.84	1.62	12
Three-Layer, 0°/90°	1.83	1.56	15
Four-Layer, 0°/90°	1.98	1.57	21
Two-Layer, ±45°	2.01	1.78	11
Three-Layer, ±45°	2.27	2.15	5
Four-Layer, ±45°	2.52	2.20	13

The ductility indexes acquired using the energy method were smaller for every test beam than the ductility indexes acquired using the conventional method. The ductility indexes for the control beam only differed by 2% proving both methods agree on ductility measurements for typical steel reinforced concrete beams. However, the difference between the ductility indexes for the strengthened beams varied by as much as 21%, showing reasonable agreement with only the three-layer, ±45° strengthened beam. This could imply the conventional ductility measurement method overestimates the actual ductility of a CFRP strengthened reinforced concrete beam due to the oversimplification of this measurement method.

Table 5.11 shows a comparison of tested ductility index of the strengthened beams to the control beam.

Table 5.11: Comparison of Tested Ductility Index of Strengthened Beams To Control Beam

Test Beams	Conventional Ductility Index	Percent Decrease w/r to Control Beam	Energy Ductility Index	Percent Decrease w/r to Control Beam
Control	2.34	N/A	2.29	N/A
Two-Layer, 0°/90°	1.84	21	1.62	29
Three-Layer, 0°/90°	1.83	22	1.56	32
Four-Layer, 0°/90°	1.98	15	1.57	31
Two-Layer, ±45°	2.01	14	1.78	22
Three-Layer, ±45°	2.27	3	2.15	6
Four-Layer, ±45°	2.52	+8	2.20	4

Both methods showed a considerable decrease in the ductility index of the 0°/90° strengthened beams as compared to the control beam, as shown in Figure 5.21. Using the conventional measurement method, the ductility index of the 0°/90° strengthened beams was reduced by 15%-22% with respect to the control beams. The energy method measured a similar reduction in the ductility index for each of the 0°/90° strengthened beams with respect to the control beam (29%-32%). However, the reduction in ductility, using either method, did not follow any pattern as more CFRP layers were applied.

In general, the ±45° strengthened beams provided a smaller decrease in the ductility index with respect to the control beam than the 0°/90° strengthened beams. Excluding the two-layer, ±45° strengthened beam (inaccurate results due to flaw in bonding process), the conventional method showed little to no reduction in the ductility index with respect to the control beam. With the energy method, and once again excluding the two-layer, ±45° strengthened beam, a ductility index reduction with respect to the control beam of only 3%-6% was measured. Also, as seen with the 0°/90° strengthened beams, the ductility index measured with either method did not follow any pattern as more CFRP was applied to the concrete beams

Table 5.12 shows a comparison of the tested and theoretical ductility index of each test beam.

Table 5.12: Comparison of Ductility Index from Test Results and Theoretical Model

Test Beams	Conventional Method			Energy Method		
	Tested	Theoretical	Percent Difference	Tested	Theoretical	Percent Difference
Control	2.34	2.54	8	2.29	2.37	3
Two-Layer, 0°/90°	1.84	2.26	19	1.62	1.87	13
Three-Layer, 0°/90°	1.83	2.16	15	1.56	1.75	11
Four-Layer, 0°/90°	1.98	2.08	5	1.57	1.65	5
Two-Layer, ±45°	2.01	2.43	17	1.78	2.11	16
Three-Layer, ±45°	2.27	2.37	4	2.15	2.05	5
Four-Layer, ±45°	2.52	2.31	8	2.20	1.98	10

The theoretical model did not accurately predict the ductility index for the test beams using either measurement method. A majority of the values were under-predicted by as much as 19% with the conventional method and 16% with the energy method. The inaccuracy can be traced to the inability of the theoretical model in predicting the deflection at the yielding of the steel reinforcement. This point is critical for the prediction of the ductility index using both measurement methods. Since it was not predicted accurately by the theoretical model, the prediction of the ductility index was also not accurate.